CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

THE UNIVERSITY OF MICHIGAN
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NOVEMBER 1959 **VOL. LIV. NO. 11**

FIFTY-FOURTH YEAR OF PUBLICATION

PRICE 2s. ANNUAL SUBSCRIPTION 24s. POST FREE. $3.90 in Canada and U.S.A.

LEADING CONTENTS

No. 624 ISSUED MONTHLY Registered for Canadian Magazine Post

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LIV, No. 11. LONDON, NOVEMBER, 1959.

RETIREMENT.

Mr. H. L. Childe, who has been Editor of this journal for thirty-eight years, is retiring on December 31 next. During this period he founded and edited the " Concrete Year Book ", the monthly journals " Concrete Building and Concrete Products " and " Cement and Lime Manufacture ", and the " Concrete Series " books of which more than 600,000 have been sold, and has also been General Manager of the business of Concrete Publications Limited.

IN the year 1922 the method of designing reinforced concrete was the same as it was when the material was introduced into this country from France more than twenty years previously, and generally it is the same to-day. This is a wonderful commentary on the ability and thoroughness of the Continental engineers of the previous century who devised a design procedure that has stood the test of some seventy or eighty years. The method of designing for conditions causing failure used on the Continent in the 1930's was incorporated in a British code of practice only two years ago, but in this method also the general procedure is still the same as in the last century. In the early 1920's most reinforced concrete design was done by a few firms of French, Danish, or U.S.A. origin. Some of these firms were more or less agents for French concerns, and some of the designs for structures in this country were actually made in France. Some of these firms made strenuous efforts to keep the work to themselves by pretending that the mathematics were so difficult that only specialists could manage them. The writer well remembers the senior partner of one such firm abusing him for publishing books which he said " enabled any borough engineer to design in reinforced concrete "—an unexpected compliment. History repeated itself in the attempts made to keep the design of " shell " roofs and prestressed concrete in the hands of a few specialists until it was discovered that the processes were not indeed tasks fit for magicians only. The offices of the " specialist " firms who started the industry in this country were the schools in which were trained men who later started their own practices, and many of whom became eminent in their profession as consultants free from the shackles of proprietary systems and contracting.

Many architects and clients still had to be convinced that concrete was a reliable material. The extensive use of concrete at the British Empire Exhibition of 1924 was a great help, for it received extensive publicity in the daily press. But it was an uphill task, and the propaganda used was sometimes amusing —for example, a popular comedian of the day was paid to introduce into his turn an episode in which he hit a man on the head and exclaimed, " My word, it's as hard as concrete, and nothing can be harder than that ! " In those days the " Daily Mail " had advertisements on its front page, and this page was frequently bought by the cement industry and used to further the interests of concrete. The " Daily Mail " then had the largest circulation of any British daily paper, but it has since been beaten by others who recognise three kinds of news, namely " news ", " HInews ", and " SAnews ", and the greatest of these is SAnews (for the benefit of the uninitiated these terms define the sort of news given in what are known as " quality " newspapers, human-interest news, and sex-appeal news). The task of promoting the use of concrete was made more difficult because of the deterioration of some of the early structures in which the dimensions had been skimped and concrete of the consistency of soup was used in order to produce the cheapest possible structures in competition with other materials—prejudice was often overcome by cheapness. Many of the early concrete structures were, however, a credit to their designers and builders, as is frequently shown in our extracts from this journal of fifty years ago.

The 1920's were a period of important development. In the U.S.A. Professor Duff Abrams made an investigation into the effect of water content on the strength of concrete and produced his water-cement-ratio curves. This confirmation of what had been known before the professor was born received much publicity throughout the world, and led to the use of lower ratios of water to cement and consequently better concrete. More attention began to be given to quality than to cheapness. Because stiffer concrete had to be compacted, shutter vibrators, followed by immersion vibrators, were imported from the Continent. Rapid-hardening Portland cement, which originated on the Continent, was made in this country and it was soon found that, due to the lower density of this more finely-ground material, less cement was used when it was measured by volume. To settle this problem an eminent engineer made extensive tests and proved that the more finely a cement was ground the bulkier it became, and that the more it was compacted the greater its density. Here again phenomena that were known before the investigator was born were re-discovered, but the publication of the results served a useful purpose in encouraging the measurement of cement by weight. A German process for making blastfurnace Portland cement was introduced about the same time, high-alumina cement was imported from France, and a white cement from the U.S.A. A variety of concrete " improvers " was imported from the Continent and the U.S.A., and gradually replaced hydrated lime which was about the only material previously added to concrete to improve its plasticity and permit the use of less water to achieve a given workability. Events have shown that, with the raw materials and the manufacturing processes in use in the cement industry, the greatest improvement in the properties of concrete are likely to be derived from the addition of chemicals, the most important of which so far is the introduction from the U.S.A. of air-entraining agents.

In the 1920's the Zeiss " shell " dome was built in Germany, and Continental engineers were experimenting with prestressed concrete. The first cast-in-situ piles in this country were introduced from Belgium; systems of forming bored piles of large diameter have since been imported from France and elsewhere. Systems of steel shutters, weigh-batching plants, continuously-moving forms, and concrete chuting plants came from the U.S.A., and a system of sliding forms

has since been imported from Sweden. There was much concern in the U.S.A. about the appearance of concrete surfaces, and it became fashionable to line shutters with plywood to produce smooth surfaces, and to use rough timber shutter-boards and wet mixtures so that the surface of the concrete would be (except for its colour) a faithful copy of unwrought timber, including all its knots and other imperfections—the results, both smooth and rough, were claimed to be " concrete expressing itself " and were called architectural concrete. To-day rough surfaces are generally produced by exposing the aggregate.

In the 1920's and 1930's new types of aggregate appeared. The coke-breeze previously used for making slabs and for reinforced floors, and which had given so much trouble (the breeze was obtained from gas-works and was the same material that was used by blacksmiths for their furnaces), was replaced by clinker from electricity works which has a much smaller combustible content. The idea of making insulating concrete by omitting the finer particles of the aggregate, and known as " no-fines " concrete, was introduced from Holland. Processes for making foamed-slag aggregate came from Germany. Expanded clay and shale aggregates and pulverised-fuel ash were used in the U.S.A., and have since been used in this country. " Gas " and cellular concretes made by the addition to cement of aluminium powder or other chemicals were developed in Denmark, Germany, and elsewhere, and were introduced into this country in the 1930's and more recently; last year a factory was built in England for making lightweight insulating concrete according to a process used in Sweden for many years. From Poland came the idea of using wood-wool and cement for making insulating slabs. New types of reinforcement and the use of chemicals to consolidate soil came from the Continent.

The production of concrete roof tiles started in this country in the 1920's. They were made one at a time on simple hand-operated machines until the first mass-production machine was imported from Germany. To-day more houses are covered with concrete tiles than with any other material. Before the arrival of automatic tamping machines and pressure machines from the Continent, and spinning machines from the Continent and Australia, pipes were made in wooden or steel moulds in which the concrete was tamped by hand. Blocks were made, generally one at a time, in metal or wooden moulds and tamped by hand. These have been almost entirely replaced by power-driven machines, and by the German process of casting blocks on a floor without pallets. The growth of the precast concrete industry has more than kept pace with the increased use of concrete in construction, for nowadays it is the exception to see any other material used for paving flags, kerbs, fence posts, lamp standards, and other features of the street scene. Methods of design have also been introduced from abroad, notably the method of moment-distribution which originated in the U.S.A. From abroad also came expanding cement and sulphate-resistant cement, and the ideas of transit-mixers, delivering loose cement in pressurised vehicles and the installation of storage silos on construction sites.

The year 1934 saw the publication of the first British code of practice for reinforced concrete. It served a useful purpose in putting on record what was considered by the Department of Scientific and Industrial Research to be good practice in design and construction, but it had two serious disadvantages. First it led, as it was intended to do, to architects and clients specifying that designs should comply with the recommendations of the code, and as it was adopted by most of the authorities with building regulations it to all intents and purposes became mandatory. It is true that a waiver could be applied for if something novel was proposed, and there was sufficient time to allow for a waiver to be

granted or refused, but generally it became a standard to be worked to, with the result that inventiveness was discouraged. Secondly it recommended thin covers of concrete to the reinforcement, which everyone else knew was the cause of nearly all the failures of early structures. In the code of 1934 it was recommended that half an inch of concrete was sufficient protection for the reinforcement in slabs exposed to the weather, with the result that some of the structures designed in accordance with the code soon had to be repaired by the gunite process that had by then been imported from the U.S.A. In the revised code issued by the British Standards Institution in 1948 the cover recommended was increased to one inch for external work and for work subjected to "particularly corrosive conditions". This was still too small, and it was not until 1957 that it was increased to one and a half inches.

The proliferation of standards issued by the British Standards Institution has also had an effect on the concrete industry, and has probably done more harm than good. Although consumers are invited to sit on the committees that prepare these standards, in practice the standards generally express the views of the trade associations concerned. The result is that, as has often been pointed out by the writer, some of them are in fact standards of mediocrity or even of inferiority. Nowadays, when most of the concrete products that are the subject of British Standards are ordered by public authorities, it makes life very easy for their engineers or purchasing departments to order everything according to a British Standard, perhaps without having read it. How different from their predecessors, who wrote their own specifications and purchased on a basis of quality and price.

To-day concrete is not merely accepted as a cheap substitute for other materials. It is recognised as the most suitable and economical material for many types of structure, and is taking the leading part in the present building boom. It would seem that, in the absence of a political upheaval or a financial crisis, the industry will prosper. Increasing demand and more mechanisation will need more factories, and more factories will need more generating stations. Better roads and buildings for parking cars are essential. Much slum property has yet to be replaced. More civic centres and public amenities, and extravagances to the glory of bureaucracy, will be demanded, if only to provide work for the huge number of architects and structural engineers on the permanent staffs of public bodies. If the present trend continues still more non-industrial buildings will be needed, for in industry generally from 1948 to 1957 the number of non-manual workers increased by 636,148 (48 per cent.) while the number of manual workers increased by 928,025 (that is only 12 per cent.). It is stated that whereas in 1926 Imperial Chemical Industries, Ltd., employed one "staff" employee to 4·1 people paid by the hour, the ratio is now 1 to 2·04; in this concern the number of staff workers has increased by 45 per cent. compared with an increase of 2 per cent. in the number of manual workers. At this rate there may soon be more non-manual than manual workers—but extrapolation is as foolish as prophecy. An indication of the material prosperity of the industry is indicated by the number of situations vacant advertised in this journal and elsewhere—in the 1920's it was an exception to find even one such advertisement in a copy of this journal. In the 1920's a designer-draughtsman was paid about £250 a year and a senior designer up to £400 a year, and the output per man per year was much greater than it is to-day. The present agitation in this country for higher salaries for members of professional engineering societies reflects a trend of "levelling" that is probably world-wide. For example, the New York correspondent of a London daily paper reports that the man who cleans his windows formerly worked 8½ hours a day for 90 dollars a week as

a technician in the cinematograph industry but now as a window cleaner works 6½ hours a day for 156 dollars a week, which is about twice the average salary of a school teacher in New York: "Who cares about pride?" asked the man now in overalls.

During 1914–1918 politicians promised a post-war land fit for heroes to live in, but no one said how this was to be brought about and the nation was quickly disillusioned. During 1939–1945 we were told that a post-war Utopia would be conjured up by scientists, but their most notable achievement so far has been to produce new means for the mass destruction of mankind, with the result that there is fear of Inferno rather than a vision of Utopia. Politicians have so arranged our affairs that there is little unemployment, more people have more goods and no one need be homeless or hungry, but many governments throughout the world must be very concerned at the certainty that quick disarmament would result in unemployment on a scale seldom known before.

The present enthusiasm for science and technology involves a risk that they may be accepted as ends in themselves, a way of life in which a man who is engaged in such pursuits is all-sufficient and has nothing more to learn. This fallacy was pointed out in this journal long before it became a serious topic in Parliament, indeed before educationists themselves became aware of it or at any rate admitted that it existed.

On the occasion of his retirement the writer would again emphasise the need for men to look beyond the mere routine of a technical occupation, to seek friends in non-technical occupations, to read and argue rather than listen to radio and television pundits and "personalities", so that they may widen their outlook, see the wood as well as the trees, discover that calculating stresses is not engineering, that testing other people's ideas or products or repeating tests already made elsewhere are not research, that the passing of an examination is not an end in itself but a mere date when a man has acquired some information that should be useful when he starts to think for himself. The inability of many scientists and technologists to express themselves clearly in their own language, or even to spell everyday words, is a serious disadvantage, not only to them but to the community. And so is their use of jargon which may mean something to them but nothing to anyone else, and which has been made worse by the publication by the British Standards Institution of a very bad Glossary of Terms for Concrete and Reinforced Concrete. Much has been written about the difficulty experienced by scientists and technologists in making themselves intelligible to others: an example in our own industry was seen the other day when a reporter in describing an exhibition of photographs of modern buildings saw one labelled "Shell Construction" and referred to it as "a building with a shell-shaped roof", leaving his readers to guess whether it resembled an oyster, a winkle, or an egg—if Alice again peers Through the Looking Glass she will find that Humpty Dumpty has many like-minded companions.

In these days, when few scientists or technologists are properly educated at school, they must go out of their way to acquire knowledge and wisdom to the end that this country may lead instead of follow the rest of the world as the concrete industry has done so far. In the 1920's and 1930's very little was spent on research in this country, and that may be one of the reasons why all the innovations and developments mentioned in these notes came from abroad. More recently vast sums have been spent on research but, as readers of this journal will know, we still depend on other countries for most if not all new developments in concrete design, materials, construction and machinery. We are told that we have "never had it so good", but we can only keep our affairs in that happy state by cultivating our brains and using them.—H. L. C.

Book Reviews.

" Statically Indeterminate Structures." By J. R. Benjamin. (London: McGraw-Hill Publishing Co., Ltd. 1959. Price 85*s.* 6*d.*)

THE basis of the method of analysing statically-indeterminate structures adopted by the author is a sketch of the shape of a loaded structural member or a complete structure. Two or more sketches may be required for one problem, each sketch being a correction of the previous one. The selection of the correct sketch is, states the author, " an act of judgement ". Having obtained a deformation diagram which expresses the most likely shape, the remainder of the solution of a problem comprises determining the points of inflection by inspection and calculating by statics the bending moments at the joints. If the statical calculation produces two or more unequal moments at a joint the average is used unless the difference be very large, in which case the deformation diagram should be altered. Although the method is approximate, the author shows that, if the assumed deformation-diagram is reasonable, the results compare well with those obtained by more rigid methods. The method is applied to continuous beams, rectangular frames with vertical and horizontal loads, mansard and gable frames, and arches. The analysis of structures stiffened by transverse walls (called shear walls) and subjected to horizontal loads due to wind, blast, or seismic forces is given in detail and is claimed to be largely original.

The method should be used with caution by inexperienced designers, who should check the results by one of the common methods until they gain confidence. The idea of visualising the deformed shape of a structure is, however, an excellent discipline for experienced and inexperienced designers alike, since there is a common tendency to think of bending moments and other effects of loading as merely mathematical expressions and not always to realise that stresses are related to the consequent elastic deformation.

" Beton in chemisch angreifenden Wässern." By Kurt Seidel. (Berlin: Wilhelm Ernst & Sohn. 1959. Price 14 D.M.)

THE author reports on tests of concrete specimens and structures exposed for many years to sea-water or water from peat. Dense concrete made with Portland cement, Portland-slag cement, or Portland-blastfurnace cement, and with cement contents of 675 lb. to 845 lb. per cubic yard, with or without the addition of trass, remained undamaged in structures after forty years' immersion in sea-water. Concrete cubes did not completely resist chemical attack in sea-water or peat-water if the concrete when mixed had a high water-cement ratio. Dense concrete, independent of the type of cement, withstood the attack of peat-water for more than thirty-five years without any significant deterioration.

" Allmän Teori für Beräkning av Armerad Betong." By Hjalmar Granholm. (Gothenburg, Sweden: Chalmers Tekniska Högskolas Handlingar. 228 pages. Price 20 Kr.)

THIS is an account of an investigation of the stresses in reinforced concrete immediately before failure. The work is printed in the Swedish language, and has short summaries in English and French.

" Keskeisesti Kuormitetun Teräsbetoniseinän Lommahduslujuus." By Arvo Ylinen. (Helsinki: State Institute for Technical Research. 1958. No price stated.)

THIS work of about fifty pages, printed in the Finnish language and with annotations in English, describes tests to determine the resistance to buckling of reinforced concrete walls with various conditions of fixity at the edges and subjected to concentric loads. One of the conclusions is that, contrary to theory, the increase in resistance due to reinforcement placed centrally in a wall is insignificant. The tests confirm the results of other investigations, namely that it is useless to provide reinforcement in the centre of a wall in an attempt to increase its load-bearing capacity.

Book Received.

" Reinforced Concrete Simply Explained." By Oscar Faber. 5th edition, revised by John Faber. (London: Oxford University Press. 86 pages. Price 12*s.* 6*d.*)

Chimney 350 ft. High at a Cement Works.

By JOHN FABER, B.Sc., M.I.C.E., M.I.Struct.E.

A REINFORCED concrete chimney about 350 ft. high (*Fig.* 1) has been completed recently for the Associated Portland Cement Manufacturers, Ltd., at Bevans Cement Works, Northfleet, Kent. The chimney is of circular cross section and tapers uniformly from an external diameter of 26 ft. 4 in. at the foundation to 13 ft. at the top.

Design.

To guard against corrosion, the topmost 15 ft. of the shaft is of 9-in. brickwork banded with mild steel straps 4 in. wide and $\frac{5}{8}$ in. thick and is capped with 8-lb. lead sheet. The brickwork is carried on a band (*Fig.* 2) at the top of the reinforced concrete shaft. The band is 6 ft. 8 in. high, which is the dimension of two standard 3-ft. 4-in. lifts of shuttering. The thickness of the reinforced concrete shaft increases uniformly from 5 in. at the bottom of the band to 8 in. at a level about 95 ft. above the ground; below this level the thickness of the shaft increases to 13 in. at the top of the foundation base. At a height of about 30 ft. above the bottom of the shaft there are three flue openings, each about 5 ft. wide and 16 ft. high. The cross-sectional area of the concrete omitted at the openings is made up by thickenings at the sides of the openings. The vertical reinforcement bars are bent to clear the openings and extend up the thickenings, which are tied into the main part of the shaft by horizontal links.

The temperature of the flue gas is normally 485 deg. F., but a maximum temperature of 800 deg. F. is allowed for in the design. The temperature of, and consequently the stresses in, the concrete shaft are reduced by an internal lining of brick. Between the lining and the shaft there is a ventilated cavity the average width of which is 6 in. The lining, which is of engineering bricks jointed with corrosion-resistant mortar, is carried on continuous reinforced concrete corbels projecting from the concrete shaft at intervals of 40 ft. in the height of the chimney. Each height of the lining is tapered to protect the corbel carrying the next height of brickwork above, but is free to expand. Ports are provided in the shaft and through the corbels to facilitate ventilation. The ports through the corbels serve also to reduce the stresses due to temperature at the local increase in cross section. The 40-ft. intervals between the corbels determined the height of the lifts of shuttering, each of which was 3 ft. 4 in., and the lengths of the vertical reinforcement bars and the position of the laps. The reinforcement is round bars of mild steel at 6 in. centres. For the lower 135 ft. of the shaft the vertical bars are $\frac{3}{4}$ in. diameter, above this level they are $\frac{5}{8}$ in., and at the top of the shaft $\frac{1}{2}$ in. diameter. The circumferential bars vary from $\frac{5}{8}$ in. diameter at the bottom to $\frac{3}{8}$ in. at the top, and have 2-in. cover of concrete.

The minimum crushing strength of 6-in. works-cubes of concrete was 3600 lb. per square inch at 28 days.

Basis of Calculations.

The calculations for the chimney were based on the following theory.* Referring to *Fig.* 3a, the notation is as follows.

* We are informed that this theory will be described fully in the second edition of "Reinforced Concrete" by Oscar Faber, which is being revised by Mr. John Faber and Mr. Frank Mead. The notes on the calculations in this article are given by permission of the publishers, Messrs. E. & F. N. Spon, Ltd.

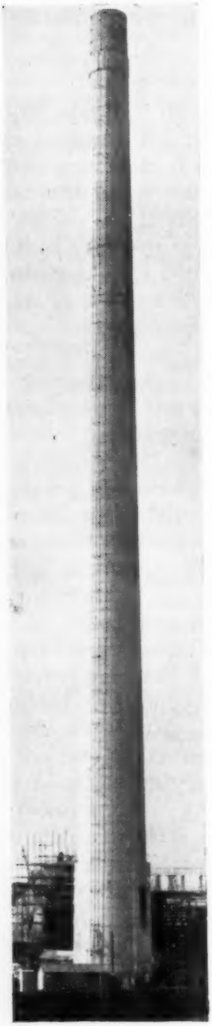


Fig. 1.

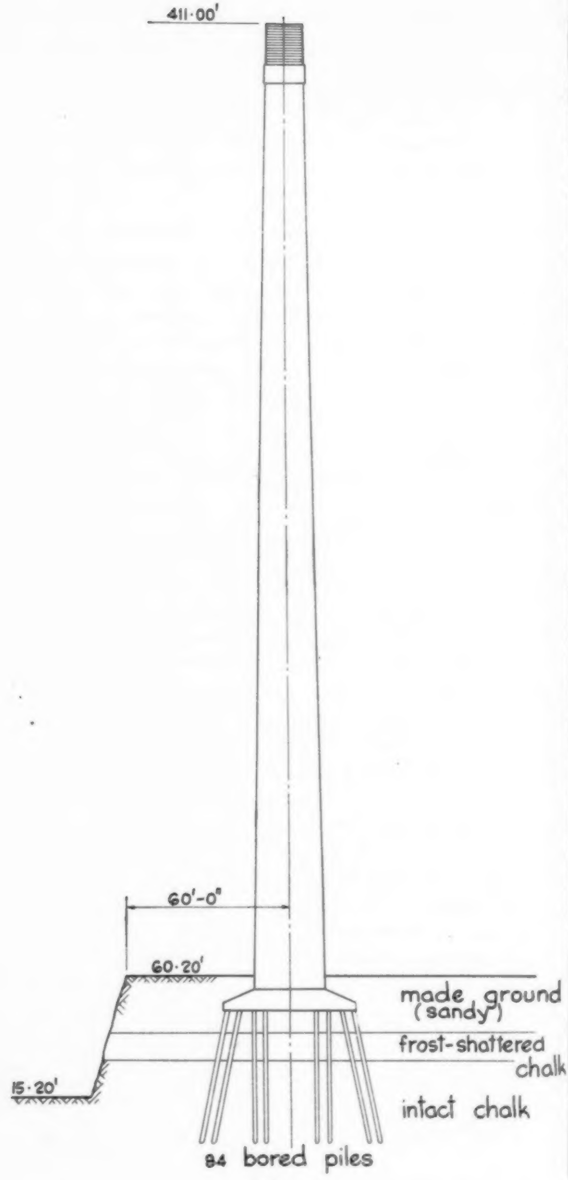

411·00'
60'-0"
60·20'
made ground (sandy)
frost-shattered chalk
15·20'
intact chalk
84 bored piles

Fig. 2.

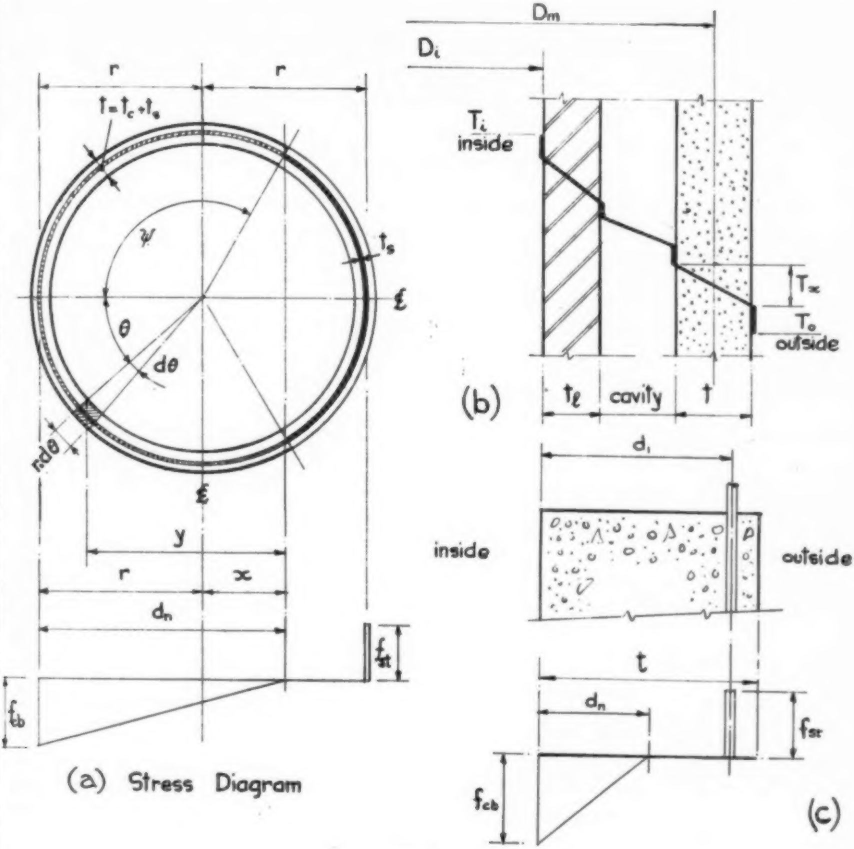


Fig. 3.

2ψ, the angle subtended at the centre, so that $\cos\psi = \dfrac{x}{r} = g$.

f_{cb}, compressive stress in the concrete.
f_{st}, tensile stress in the vertical reinforcement.
C, total compression.
T, total tension.
M_C, total moment due to total compression.
M_T, total moment due to total tension.
t_s, thickness of an annular ring of area equal to the area of the vertical reinforcement.
t_c, equivalent net thickness of the concrete shaft.
t, actual thickness $(t_s + t_c)$ of shaft.
r, mean radius of shaft.
A_{st}, total cross-sectional area of vertical reinforcement.
k_1, k_2, k_3, k_4, constants dependent only on the value of k.

Also W, total weight of chimney above any horizontal plane.
M, applied moment at that plane.
e, eccentricity $\left(\dfrac{M}{W}\right)$.

The angle ψ is determined by the ratio $k = \frac{f_{st}}{f_{cb}}$, and it can be shown that

$$C = k_1 r f_{cb}(t_c + mt_s) \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$M_C = k_2 r^2 f_{cb}(t_c + mt_s) \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$T = k_3 r m f_{cb} t_s \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$$M_T = k_4 r^2 m f_{cb} t_s \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

If $\bar{y}$ is the distance from the centre of compression to the neutral plane, and l_a is the lever arm between the centre of compression and the centre of tension, it follows, by equating moments about the centre of compression, that

$$M = W(\bar{y} + x) + Tl_a,$$

in which $\quad l_a = \frac{M_C}{C} + \frac{M_T}{T} = r\left(\frac{k_2}{k_1} + \frac{k_4}{k_3}\right) \quad$ and $\quad \bar{y} = \frac{M_C}{C} = r\frac{k_2}{k_1}$.

Substituting for T from equation (3) and rearranging,

$$t_s = \frac{A}{2\pi}\cdot\frac{M}{r^2 f_{cb}} - \frac{B}{2\pi}\cdot\frac{W}{r f_{cb}} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

in which $\quad A = \frac{2\pi k_1 k_3}{mk_3(k_3 k_2 + k_1 k_4)} \quad$ and $\quad B = A\left(\frac{k_2}{k_1} + g\right)$.

Since $A_{st} = 2\pi r t_s$,

$$A_{st} = A\frac{M}{r f_{cb}} - B\frac{W}{f_{cb}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Also, equating forces, $W = C - T = r f_{cb}[k_1 t_c + m(k_1 - k_3)t_s]$

from which $t_c = \frac{W}{k_1 r f_{cb}} - m\left(\frac{k_1 - k_3}{k_1}\right)t_s$. Substituting for t_s from equation (5)

and rearranging, $t_c = k_5\cdot\frac{W}{r f_{cb}} - k_6\frac{M}{r^2 f_{cb}}$, in which

$$k_5 = \frac{(2\pi + k_1 B)(k_1 - k_3)m}{2\pi k_1^2} \quad \text{and} \quad k_6 = \frac{m(k_1 - k_3)A}{2\pi k_1}.$$

Since $t = t_c + t_s$, the total thickness of the shell is given by

$$t = D\frac{M}{r^2 f_{cb}} + F\frac{W}{r f_{cb}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

in which $D = \frac{A}{2\pi}k_6$ and $F = k_5 - \frac{B}{2\pi}$.

Equation (7) may be rearranged in the form $\frac{M}{tr^2 f_{cb}} = \frac{1}{\left(D + \frac{F}{e/r}\right)}$, and since D and F are dependent on the value of k, this gives a relationship between $\frac{M}{tr^2 f_{cb}}$ and $\frac{e}{r}$ for various values of k. Also, since the percentage of vertical reinforcement is given by $r_p = \frac{A_{st}}{2\pi rt} \times 100$, it follows from equations (6) and (7) that

$$r_p = \frac{100}{2}\left(\frac{A - \dfrac{B}{e/r}}{D + \dfrac{F}{e/r}}\right),$$ so that similarly, for various values of r_p, there is a relationship between $\dfrac{M}{tr^2f_{cb}}$ and $\dfrac{e}{r}$. Curves can therefore be drawn from which k and r_p can be read directly for any chosen cross section if $\dfrac{M}{tr^2f_{cb}}$ and $\dfrac{e}{r}$ are known.

The total difference of temperature through the brick lining and the concrete shaft is shown diagrammatically in *Fig.* 3b. There are five stages in the fall of temperature from the flue-gas temperature T_i to the external-air temperature T_o, namely, (*a*) surface effect on the inside of the lining, (*b*) fall of temperature through the lining, (*c*) double surface effect at the cavity and effect of ventilation of the cavity, (*d*) fall of temperature through the concrete shaft, and (*e*) surface effect on the outside of the shaft.

If the quantity of heat transferred at stage (*a*) be denoted by Q, then the heat transferred at stage (*b*) must also be Q; and if $(1 - r)Q$ is the heat drawn up the cavity by ventilation, the balance rQ will be the quantity transferred at stages (*c*), (*d*), and (*e*). The quantity Q can be expressed in the form

$$Q = U(T_i - T_o) \qquad (8)$$

in which T_o is the temperature of the flue gas, T_o is the temperature of the air outside the chimney, and U is the overall transmission in British thermal units per square foot per degree Fahrenheit per hour, so that

$$\frac{1}{U} = \frac{1}{K_1} + \frac{t_l}{C_1} + r\left(\frac{1}{K_c} + \frac{t}{C_2} + \frac{1}{K_2}\right) \qquad (9)$$

in which K_1 and K_2 are transmission coefficients for the inner and outer surfaces of the shaft, K_c is the transmission coefficient for the cavity, C_1 and C_2 are conductivity coefficients per inch thickness of the lining and the shaft respectively, and t_l and t are the thicknesses in inches of the lining and the shaft.

Considering the concrete shaft alone,

$$rQ = \frac{C_2}{t}.T_x \qquad (10)$$

in which T_x is the difference of temperature through the shaft. From equations (8) and (10), it follows that $T_x = r\dfrac{t}{C_2}(T_i - T_o)U$, and, allowing for the increase in the cross-sectional area of the shaft due to the mean diameter of the shaft being greater than the internal diameter,

$$T_x = r\frac{D_i}{D_m}\cdot\frac{t}{C_2}(T_i - T_o)U \qquad (11)$$

in which D_i is the internal diameter of the chimney and D_m is the mean diameter of the concrete shaft.

The difference of temperature through the shaft results in tensile stresses at the outer face and compressive stresses at the inner face. Ignoring stresses from other causes, there will be a neutral plane within the thickness of the shaft.

Fig. 4.—Foundation of Chimney.

If it is assumed that the concrete cracks, leaving the reinforcement to resist the whole of the tension, the conditions are as shown in *Fig.* 3c. If T_{x1} is the difference of temperature of the reinforcement and the inner surface of the concrete, $T_{x1} = \frac{d_1}{t}T_x$, and then the temperature difference between the neutral plane and the inner surface of the concrete is $T_{x1}\frac{d_n}{d_1}$, or $T_{x1}n_1$. Also, since stress = strain × E it follows that

$$f_{cb} = \alpha T_{x1} n_1 E_c, \qquad (12)$$

and

$$f_{st} = \alpha T_{x1}(1 - n_1) E_s, \qquad (13)$$

in which α is the coefficient of thermal expansion of concrete, which is the same as the coefficient of expansion of steel.

Foundation.

The chimney is 60 ft. from the edge of a cliff 45 ft. high which is the edge of an old quarry (*Fig.* 2). The upper 20 ft. of the ground is sandy and is mainly filling. Below this stratum is chalk, though the upper 10 ft. of the chalk stratum has been shattered by frost to varying degrees. Below the stratum of shattered chalk, the ground is intact, but not hard. It was decided to provide piles in this lower stratum; the total length of each pile is about 50 ft. and they extend 15 ft. or more below the floor of the old quarry. As it was not possible to bring a pile-driving frame on to the site, bored piles were used. The piles were arranged in four concentric circles, the diameter of the circles and number of piles in each circle being: 18 ft., 12 piles: 27 ft., 18 piles: 36 ft., 24 piles: and 45 ft., 30 piles. The total number of piles is 84. The piles in the outer two circles are at a rake of 1 in 5. Thus under eccentric loading due to wind the more heavily loaded

piles on the leeward side obtain support over a greater area of the better ground at a greater depth. The horizontal components of the loads on the piles are available to resist the shearing force due to the wind.

The load on each pile due to the wind was calculated by treating the piles in each circle as an annular ring of unit thickness, giving the second moment of area about the chimney axis equal to $\Sigma\pi(\text{radius})^3$. The maximum load on the leeward side is 49·4 tons per pile and the minimum load on the windward side is 0·6 ton per pile. None of the piles is in tension. The piles are 16 in. diameter and are reinforced with five $\frac{5}{8}$-in. bars. Two of the piles were tested by being loaded to 105 tons. One pile depressed $\frac{1}{8}$ in., and recovered about two-thirds of this depression when unloaded. The other pile depressed only about half this amount.

The pile-cap is 48 ft. diameter and is generally 8 ft. thick at the middle and tapers to 2 ft. 6 in. at the edges. It is reinforced in the bottom face only by a mat of $1\frac{1}{4}$-in. bars at 4 in. centres in both directions. The pile-cap was cast in four equal quadrants, with tongue-and-groove rebates at each construction joint (*Fig.* 4).

Construction.

The concrete shaft was constructed in lifts of 3 ft. 4 in. each and climbing shuttering was used. There were no vertical construction joints. The shutters were of plywood and produced a smooth face on the concrete. The linings of the shutters were renewed when about half the shaft had been constructed. The taper on the chimney was achieved by making each panel of shuttering with a slight taper. Only at mid-height is the shaft truly circular. Below and above mid-height the form of the shaft is a series of cusps. At the bottom the cusps form slightly re-entrant intersections on the external face whereas at the top the intersections produce slight arrises. These effects are not generally perceptible. The shutters were made in the first instance to suit the lowest lift of the shaft, and at each subsequent lift a narrow strip was removed from one side of each panel. Scaffolding was provided internally only, and incorporated an electrical hoist.

The chimney is provided with lightning conductors and a galvanised steel ladder with a removable bottom length.

Piling began in November 1957 and was completed in March 1958. The construction of the superstructure began immediately and the chimney was completed in June 1959. The consulting engineers were Messrs. Oscar Faber & Partners. The main contractors were Messrs. Bierrum & Partners, Ltd., and the piles were constructed by Holmpress Piles, Ltd.

Beams with Tensioned Wires and Mild Steel.

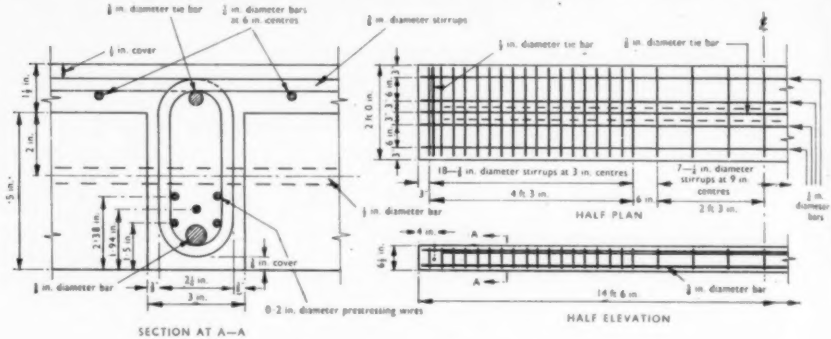


Fig. 1. Details of Type I Beams.

A BEAM designed by the late Mr. F. J. Samuely has been tested by the Cement and Concrete Association. The beam (*Fig.* 1) was 14 ft. 6 in. long and comprised a precast rectangular part 5 in. deep and 3 in. wide and a 1½-in. flange 2 ft. wide cast in place. The precast part was prestressed by five pre-tensioned 0·2-in. wires and contained a ⅝-in. mild-steel bar, the purpose of which is to replace the flange of an inverted tee-beam up to the working load. Resistance to failure is also increased, and therefore the advantage of an inverted tee-beam is obtained with a saving of shuttering.

Load tests were made on three beams and on a similar beam but without the ⅝-in. bar. A typical load-deflection curve is shown in *Fig.* 2. The dead load plus the working load is equal to half the calculated load causing failure. Mechanical strain-gauges indicated that at the working load the maximum compressive stress in the concrete of the flange was 1350 lb. per square inch. The greatest width of crack at the working load measured with the aid of a microscope was 4×10^{-3} in. The strains in the mild-steel bar were measured by electrical-resistance strain-gauges bonded to parts of the bar which had been cleaned. The strains indicated that the bar had an initial pre-compression of 13,500 lb. to 18,000 lb. per square inch. The ultimate-load tests indicated that the full calculated ultimate moment of resistance of the beam was developed when the load was uniformly distributed, whereas a beam with pre-tensioned 0·2-in. wires and subjected to concentrated loads may fail due to slipping of the wires at 80 per cent. of the calculated ultimate load.

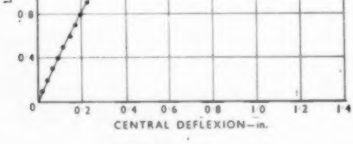


Fig. 2.

Lectures on Highway Engineering.

THE Cement and Concrete Association has arranged for the following lectures to be given at Friends' House, Euston Road, London, N.W.1, on November 17: Techniques of Concrete Road Construction in Switzerland, by Herr W. Schüepp, Road Planning and Construction in Berlin, by Herr R. Schwedler, and Traffic and Highway Engineering, by Professor M. Feuchtinger. Tickets may be had from the Association, 52 Grosvenor Gardens, London, S.W.1.

Precast Flat-slab Floors in Poland.

SEVERAL warehouses built in Poland and consisting entirely of precast concrete members are described in "Biuletyn Techniczny", No. 4, 1958. The system used for their construction comprises precast columns with ribbed heads and square or octagonal floor panels, which form a structure similar in character to a flat slab.

Typical details are shown in the illustrations. The columns are cast in single-story heights (10 ft. 6 in. in the case of the ground floor and 10 ft. in the upper floors), and near the top of each a seating is provided upon which rests the ribbed head. From each head project four diagonal arms, triangular in elevation, with a hollow square centre. The column on which the head is placed occupies one-half of this space, and after it is assembled, plumbed, and wedged in position the floor units are placed in position. Octagonal units span between the columns and a square unit is placed in the centre of each floor panel, as shown in *Fig.* 1. The square units are provided with steel dowel pins at the corners, which are inserted in holes drilled in steel plates on the ribs of the column heads, thereby ensuring that the columns are truly vertical. The columns for the next story are then erected, plumbed, and wedged, and the floor is covered with a layer of concrete cast in place. It is claimed that the result is a monolithic system which can withstand large vertical and horizontal loads.

The walls comprise two types of precast units, the widths of which correspond to those of the column-heads and of the spaces between the heads. The area of each square floor unit is one-third of the area of the bay in which it is placed; the dimensions of the units are therefore as shown in *Fig.* 1.

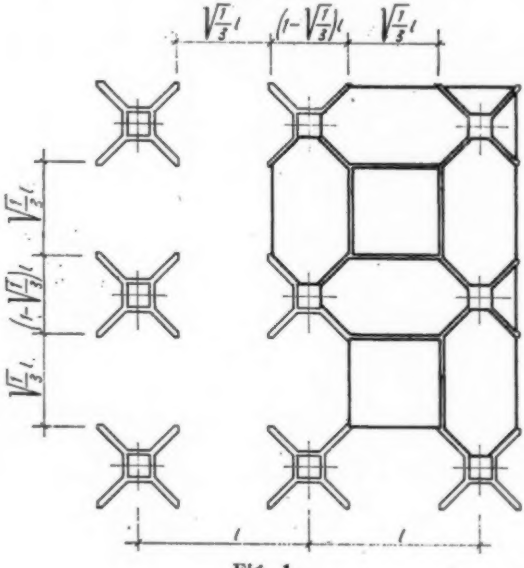


Fig. 1.

Two variations of the system have been used. In the first, which was used for a corn warehouse (*Fig.* 3) with a capacity of 1475 tons, the columns are at centres of 16 ft. 4 in. in both directions, and the floors are designed to support a live load of 300 lb. per square foot. The units are as shown in *Fig.* 4; the ribbed heads weigh 1·25 tons, the square floor units 1·65 tons, and the octagonal floor units 1·8 tons. Triangular units were used near the walls. Details of the connections of

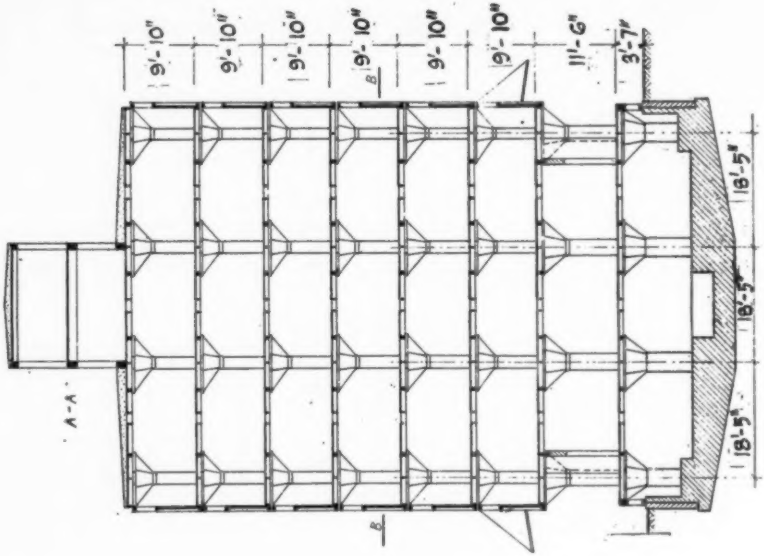

9'-10"
9'-10"
9'-10"
9'-10"
9'-10"
9'-10"
11'-6"
3'-7"
18'-5"
18'-5"
18'-5"
A-A
B
B

Fig. 3.

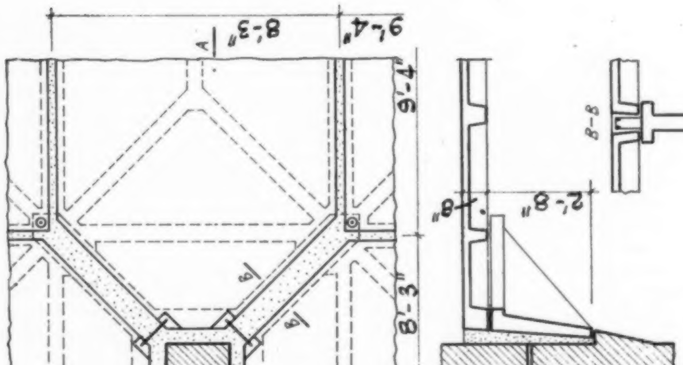

8'-3"
9'-4"
9'-4"
8'-3"
A
A
B
B
B-B
A-A
2'-8"
8"

Fig. 2.

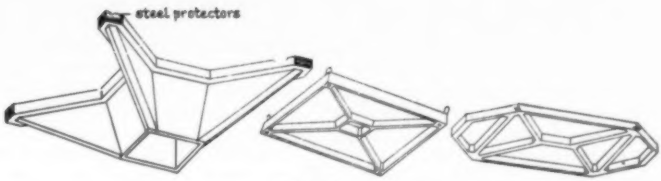


Fig. 4.

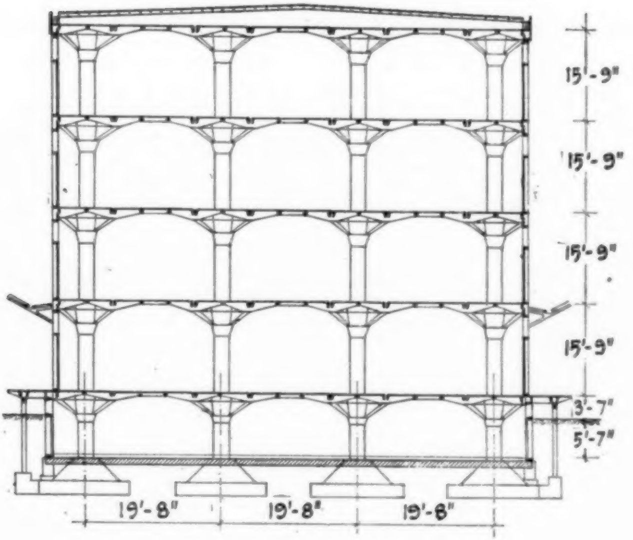


Fig. 5.

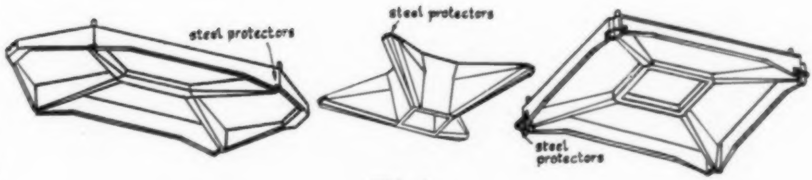


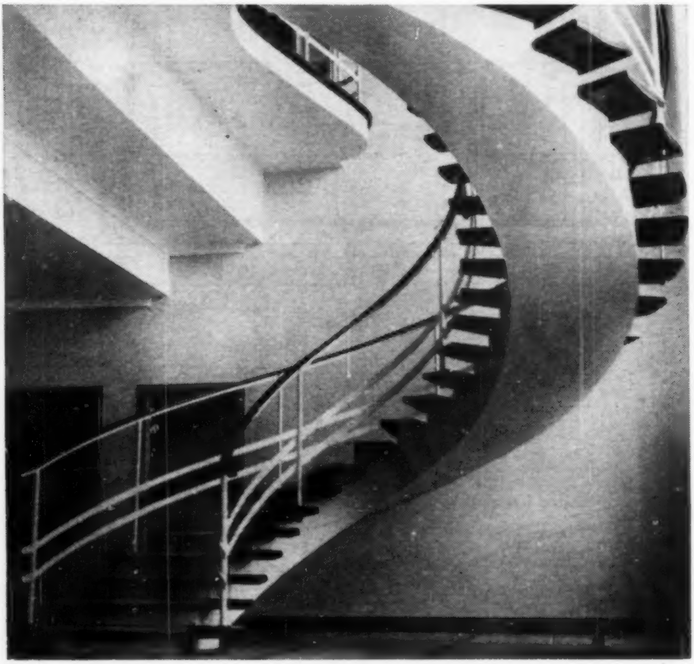
Fig. 6.

the columns are shown in *Fig.* 2; dowel bars were used to ensure the accurate positioning of the columns, and the gaps between the columns and the ribbed heads were caulked with stiff cement mortar to obtain a monolithic joint. The staircases are also of precast concrete. The precast wall units weigh 1·25 tons and 1·45 tons; only eight types of units were used for the complete structure, and were erected by a crane with a capacity of 145 tons-ft. and a working radius of 60 ft.

The second variation was used for the warehouses shown in *Fig.* 5. In this case the columns are at centres of 19 ft. 8 in. in both directions and the floors are designed to support a live load of 250 lb. per square foot. The shapes of the units (*Fig.* 6) and the method of erection are similar to those previously described, but the floor loads are supported by arch action. The unbalanced thrusts of the arches are resisted by post-tensioned pre-stressing cables in the floor covering.

Helical Staircase with Wooden Treads.

THE staircase illustrated has been built for Messrs. J. F. Robertson, Ltd., at their offices at Dundee. Connecting the ground floor with the first floor, the stairs consist of timber treads cantilevered from a helical beam 2 ft. 6 in. wide by $7\frac{1}{2}$ in. minimum depth increasing to 9 in. at the connections with the floors. The beam is designed as a spring, the lower end of which is rigidly held by a concrete block 6 ft. by 4 ft. 6 in. by 2 ft. 3 in. deep. Mr. William M. Wilson was the architect and the British Reinforced Concrete Engineering Co., Ltd., were the engineers.



A Doubly-curved Roof at Haifa.

THE Winston Churchill Auditorium (*Figs.* 1 and 3) of the Technion-Israel Institute of Technology in Haifa is about 80 ft. square in plan, and is covered with a shell roof which is curved in two directions. The shape of the roof is that generated by a parabola sliding along identical parabolas positioned along two parallel sides of the square. The translational shape so produced is the same as the rotational shape of a paraboloid of revolution.

This geometrical property simplified the shuttering, as all the supporting trusses (which were erected at 12 ft. centres) were identical. No waste of timber occurred as boards with parallel sides were used throughout (*Fig.* 2); no wedge-shaped strips were necessary, and the large radius of the roof made it possible to use straight planks of 1 in. thickness. The deviation of the paraboloid from a truly spherical surface was less than 1½ in.

Diagonal ribs, 8 in. wide and 12 in. deep, were provided in order to stiffen the roof and to support the uneven loads imposed by a false ceiling and air ducts. Special fittings were inserted at each intersection of the ribs (*Fig.* 4) to support these loads; they do not project below the surface of the concrete and therefore did

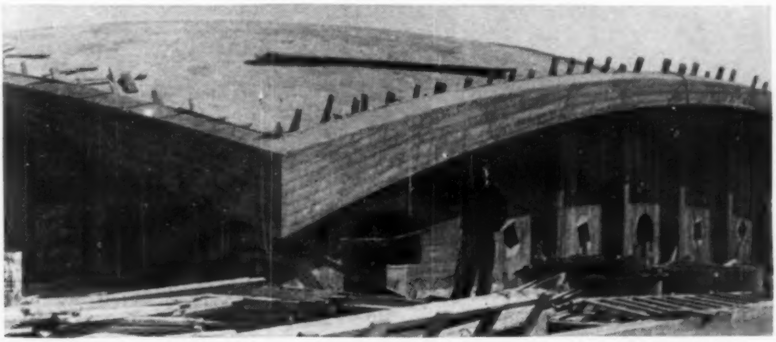


Fig. 1.

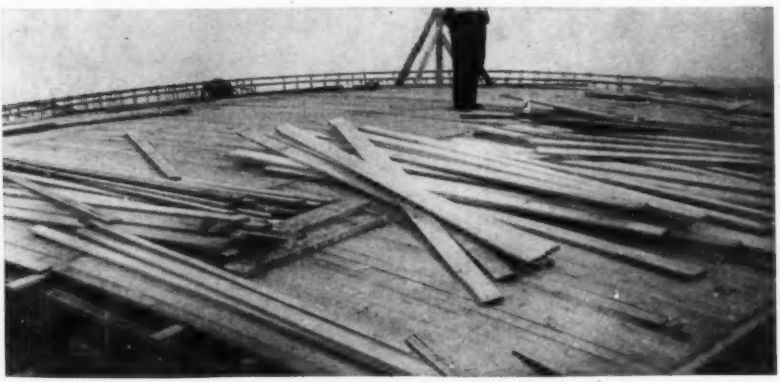


Fig. 2.—The Shuttering.

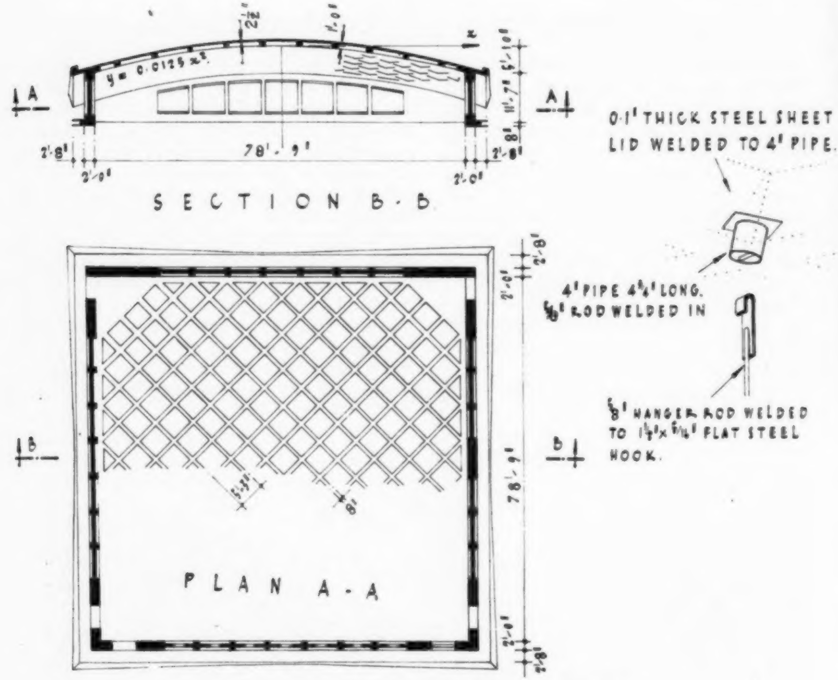


Fig. 3.—Plan and Details of Hangers.

Fig. 4.—Intersection of Ribs with Fitting for Hanger. (*See also Fig.* 3.)

Fig. 5.—Porous Board "Boxes" for Ribs.

Fig. 6.—Concreting at a Corner.

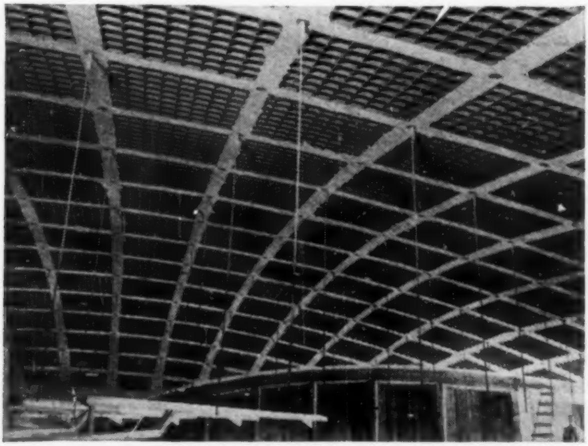


Fig. 7.—Underside of Roof.

not obstruct the shuttering. The thickness of the slab between the ribs is $2\frac{1}{2}$ in.

The hollows between the ribs were formed by boxes (*Fig.* 5), made of porous board $\frac{1}{2}$ in. thick and stiffened by transverse partitions of the same material, which were left in position. They are slightly rhomboidal (being square in plan) and their shapes differ according to their position in the roof.

The cement content of the concrete is 550 lb. per cubic yard ; the reinforcement is mild steel. A view during construction is given in *Fig.* 6 and the underside of the completed roof is shown in *Fig.* 7.

The architects are Mr. Arieh Sharon and Mr. B. Idelson, and the consulting engineer is Mr. A. N. Weidenfeld, all of Tel Aviv. The contractors were Messrs. Solel Boneh, Ltd., of Haifa.

Elevated Roads.

MANY of the motorways and road improvements in course of construction or proposed in various parts of the world include elevated concrete structures to carry roads over roads as well as over rivers and railways. The reinforced and prestressed concrete viaduct in Brussels (*Fig.* 1) is 1640 ft. long and forms part of the old inner ring-road. A motorway is being constructed through Düsseldorf and in *Fig.* 2 is illustrated a roundabout which carries the new motorway over an existing motorway. The construction of the Circular Quay Roadway in Sydney, New South Wales, Australia, is almost completed and forms part of a project comprising four radial express motorways linked by a ring-road; a crossing over an existing main street is shown in *Fig.* 3 in which is also seen the entrance to a tunnel on one of the express motorways.

Notable new schemes in Great Britain include an elevated roadway at Hammersmith (London), an orbital road at Cardiff, the Queenhill bridge over the river Severn, several bridges including one over the river Thames in connection with the Maidenhead by-pass road, and a viaduct over the river Don on the Doncaster by-pass road. Other schemes in and around London in which reinforced concrete retaining walls, bridges, and other structures predominate include bridges to carry existing cross roads over the

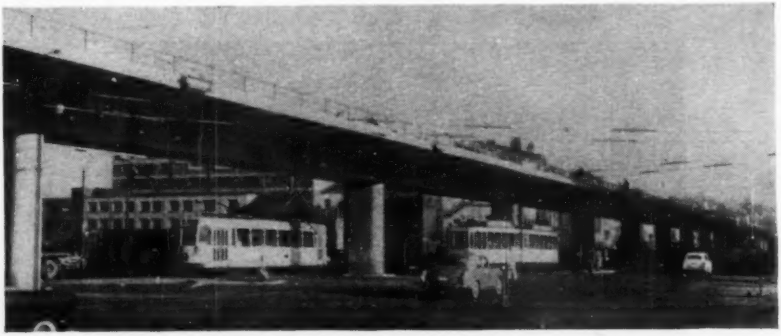


Fig. 1.—Elevated Road at Brussels.

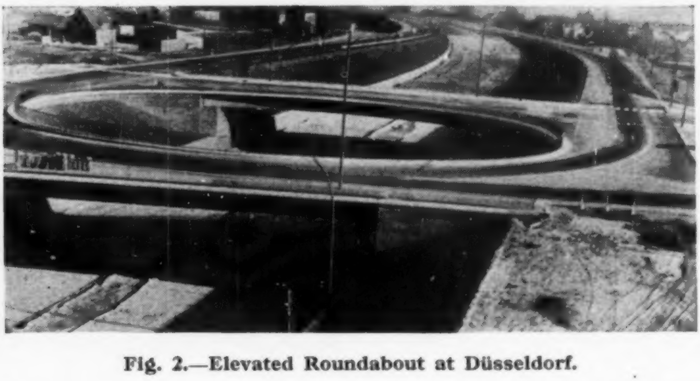


Fig. 2.—Elevated Roundabout at Düsseldorf.

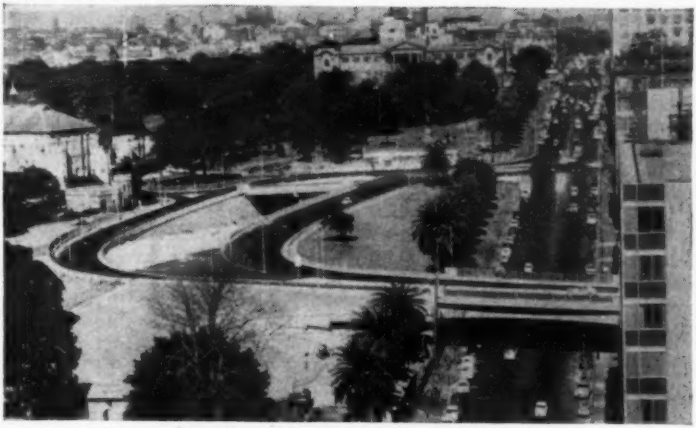


Fig. 3.—Elevated Road at Sydney.

Fig. 4.—Elevated Road at Hammersmith.

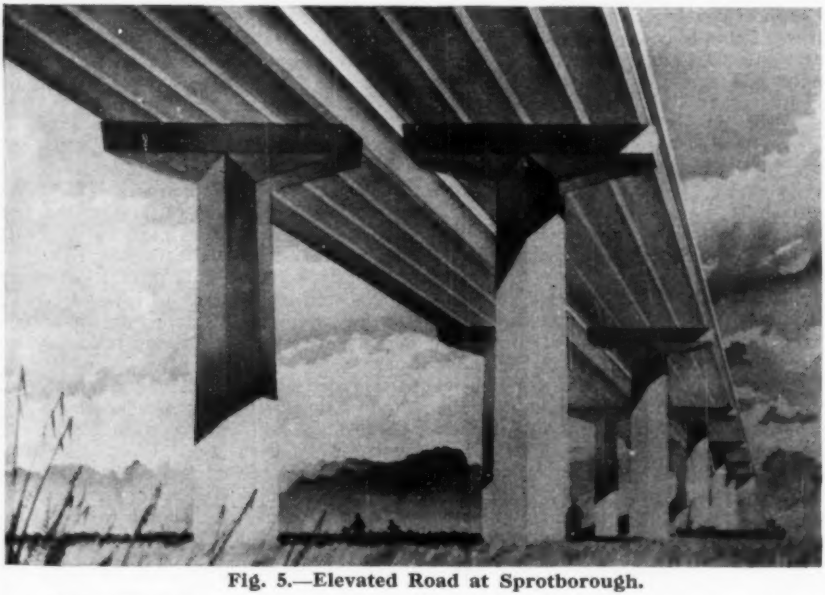


Fig. 5.—Elevated Road at Sprotborough.

Kingston by-pass road and Western Avenue, and tunnels at Hyde Park Corner and at the Elephant and Castle crossing in South London.

The elevated road (*Fig.* 4) at Hammersmith, which will form part of the extension of Cromwell Road, will be about half a mile in length and will accommodate two lines of traffic in each direction. The road will be cantilevered from a central beam carried on thirteen tapered columns spaced at intervals of up to 140 ft. The structure will be mainly of prestressed concrete, and the engineering works are estimated to cost about £1,200,000; the construction period will be about 21 months after the site becomes available. Each carriageway will be 24 ft. wide; the overall width of the structure will be 61 ft. and will provide a clearance of 16 ft. 6 in. Electrical heating elements will be embedded in the approach ramps to prevent ice forming on the surface. Precast construction, and working during periods of least traffic, are expected to reduce interference with traffic during construction. The consulting engineers are Messrs. G. Maunsell & Partners in association with the London County Council.

The Doncaster by-pass road will be seventeen miles long and will include twenty-six bridges the longest of which will be over the river Don at Sprotborough. This structure (*Fig.* 5) will be 683 ft. long and 70 ft. high and there will be two separate viaducts each of six spans. The construction will be a reinforced concrete deck carried on longitudinal steel girders supported on reinforced concrete piers and abutments. The design has been prepared by Mr. M. Lovell, O.B.E., County Engineer of the West Riding of Yorkshire.

Stabilisation of Soil by Electro-osmosis.

To support the piers and abutments of a bridge in Ontario, Canada, it was necessary to drive piles through a stratum of saturated silt 70 ft. to 130 ft. thick overlying rock. When short piles were driven to support constructional equipment, the silt was found to be dangerously unstable. Tests showed that the amount of water in the soil was 26 per cent. of its dry weight, that when the water content exceeded 24 per cent. minor vibrations caused the soil to slide, and that a reduction of the moisture-content to about 23 per cent. would enable work to proceed.

Electro-osmosis was used to stabilise the soil. The electro-osmotic coefficient of permeability was 0·000015 cm. per second per volt per centimetre, and the conductivity of the silt was 0·0003 mhos per centimetre. Tests showed that sufficient tension would develop in the water within seven days. With a potential gradient of 0·3 volt per centimetre, the tension was about 8 lb. per square inch and provided an added factor of safety against slipping.

The site was covered with 3 ft. of sand and gravel as a protection from frost. Transverse rows of anodes and cathodes 40 ft. long were installed at intervals of about 50 ft. on the slope, and a row of anodes 80 ft. to 120 ft. long and a row of cathodes 100 ft. to 140 ft. long were installed at the top and bottom of the slope. The electrodes were 6 ft. to 10 ft. apart across the face of the slope. Between 100 and 150 volts were applied to the electrodes, and the water that collected at the cathodes was removed by pumps which operated continuously. After three months the water in the soil was reduced by about 3½ per cent. and the average weight of the soil had increased by about 6 lb. per cubic foot. The water level was lowered by 40 ft. near the top of the slope and 45 ft. near the bottom. At the site of one of the piers near the top of the slope an excavation 50 ft. deep was made and the sides, which sloped at 45 deg., were not disturbed by the severe vibration caused by driving piles for the pier.

The consulting engineers were the Foundation of Canada Engineering Corporation, Ltd., the contractors were Barnett-McQueen Co., Ltd., and the electro-osmotic system was designed and made by Griffin Wellpoint Corporation. These notes are abstracted from "Engineering News-Record" for April 16, 1959.

Lift-slab Construction for a Tall Building.

THE illustrations show the use of the lift-slab method of construction in Sydney, Australia, for a building which is to be 210 ft. high and of eighteen stories. The building is L-shaped and in the angle formed by the junction of the two arms a steel frame is being erected the height of the building and braced to reduce horizontal movements which would cause difficulties in lift-slab construction. The floors from the sixth to the fifteenth stories are identical, and each floor comprises two slabs each having an area of about 7500 sq. ft., an average thickness of 10 in., and a weight of about 400 tons. The two slabs are joined by a strip about 3 ft. wide of concrete cast in place.

The slabs are cast one above another at basement level and are lifted by twenty-seven hydraulically-operated jacks (*Fig.* 2)

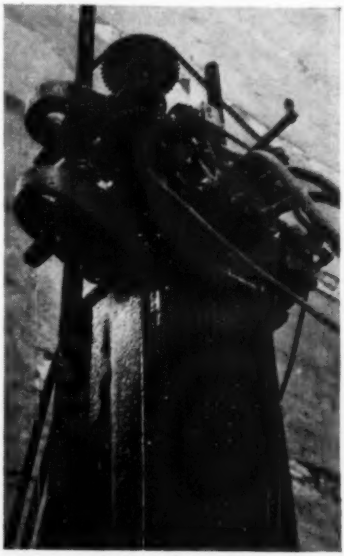


Fig. 2.—Jack at Top of Column.

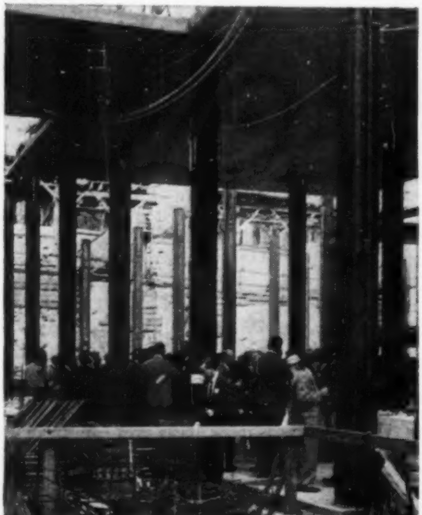


Fig. 1.—One Slab Partly Raised.

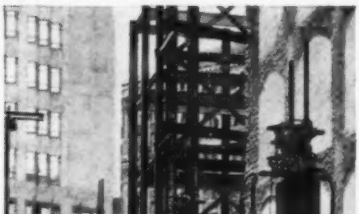


Fig. 3.—Jacks and Central Steel Structure.

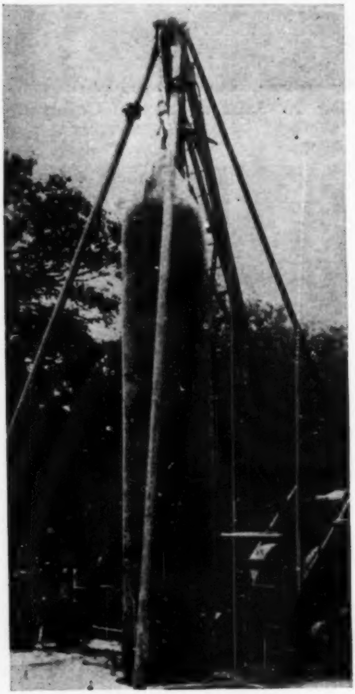
each having a capacity of 70 tons. The jacks are connected in series by the oil-supply lines (*Fig.* 3) and are fixed to the steel columns. The slabs are lifted at a rate of 4 ft. per hour by means of threaded bars turned by the oil motors. In *Fig.* 1 one slab is shown raised to the tops of the columns where it is supported while the jacks are used to raise another slab from the stack seen at the bottom of the picture.

As each slab reaches its final position it is fixed to the braced steel structure, and further precast concrete beams are placed between the columns along the façades to stiffen the structure.

It is estimated that the lift-slab method will effect a saving of about one-quarter of the cost of construction by other methods and that the time of erection will be reduced by about one-third. The architects are Messrs. McConnel, Smith & Johnson, the structural engineers are Messrs. Woolacott, Hale & Bond, and Lift Slab of Australia Pty., Ltd., are responsible for the lifting of the slabs.

Inflated Tubes for Forming Shafts.

THE illustration shows the use of an inflated rubber tube for forming a shaft of 3 ft. diameter at a gypsum mine in France. Due to the nature of the soil the shaft was constructed in lengths from 6 ft. to 9 ft. The procedure was to excavate the soil to the required depth and diameter, lower the tube and inflate it, and place the concrete between the tube and the soil. When the concrete had hardened sufficiently the core was deflated and removed, and the process repeated at the lower levels.



An Early Concrete Building in London.

AN office building known as Nos. 40–66 Broadway, Westminster, London, erected about ninety years ago by Drake's Patent Concrete Building Company and recently demolished, and which is thought to have been the first concrete building of this nature erected in London, has been the subject of an investigation by the Architect's Department of the London County Council. The building, which comprised five stories and a basement and had chimneys extending 18 ft. above the roof, was constructed of unreinforced concrete. The walls were 1 ft. 8 in. thick throughout their height, and had ½ in. of rendering on the outside and 2 in. of plaster on the inner side. The floors were of filler-joist construction, with 6½-in. by 2-in. wrought-iron joists 20 ft. long at 4 ft. 6 in. centres. The roof was slated.

A report on the concrete states: "The sample was somewhat variable in composition, being generally hard but with some friable portions. The aggregate contained besides stone, some coal, coke, wood, shells, seeds, hair, and straw, and some voids were present." The concrete appeared to be composed of about 1 part of Portland cement to 6 parts of mixed aggregate. The grading agreed closely with that given in the British Standard for 1½-in. stone.

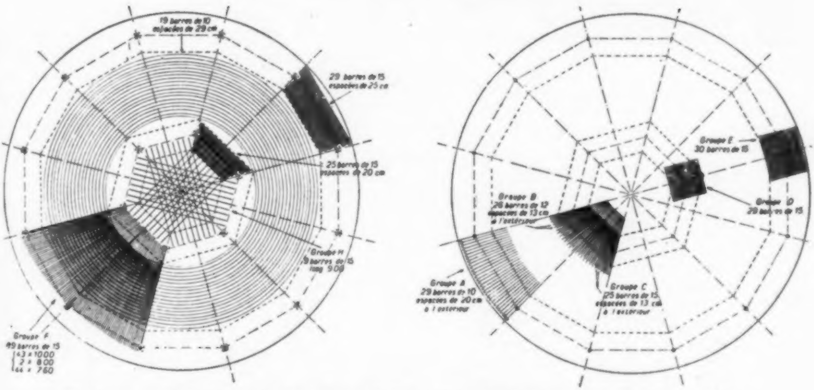
Circular Lift-slabs for Roofs.

AN unusual arrangement of buildings has been used for a school at Old Saybrook, Connecticut, U.S.A., together with an unusual method of construction. *Fig.* 1 shows a model of the school; each of the circular buildings accommodates five or six classrooms (*Fig.* 3).

The circular buildings are each 100 ft. in diameter and 9 ft. 4½ in. high from floor to ceiling. The upper slab, 10 in. thick, is supported on eighteen columns of which twelve are on a circle of 90 ft. diameter and six are on a circle of 33 ft. 2 in. diameter. The columns of the outer row are square and are formed by welding two angles; the inner columns are tubular. The ground-floor slab is 10 in. thick and, together with a beam under the outer row of columns, acts as a foundation.

For each building the roof slab was cast directly on the ground-floor slab, but separated from it by a layer of paraffin wax. Steel collars embedded in the roof slab embrace the columns. Top and bottom reinforcement is provided in the roof slab, which weighs 500 tons. *Fig.* 2 shows the arrangement of the reinforcement. When the slab had attained a sufficient strength, generally ten to fourteen days after casting, it was raised by nine hydraulic jacks fixed to the tops of nine columns. The nine jacks were

Fig. 1.—Model of School Buildings.



(Bottom reinforcement shown on left; top reinforcement shown on right.)

Fig. 2.—Reinforcement of a Roof Slab.

operated from a single electric control panel and raised a slab at a rate of 4 ft. to 5 ft. per hour ; that is, a slab was raised to its final height in about two hours. When the desired height was achieved the collars were welded to the columns, after which the jacks were removed.

It is stated that in comparison with a similar school built in the same State in the year 1950 the cost of construction was reduced by 14 per cent. by this method. The architect was Mr. Warren H. Ashley, and the foregoing is abstracted from the Belgian journal "La Technique des Travaux".

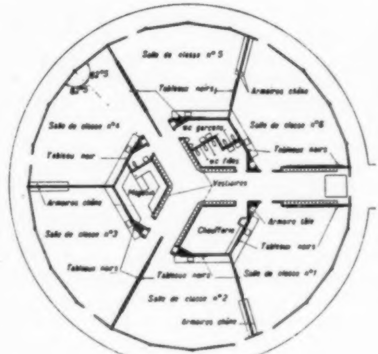
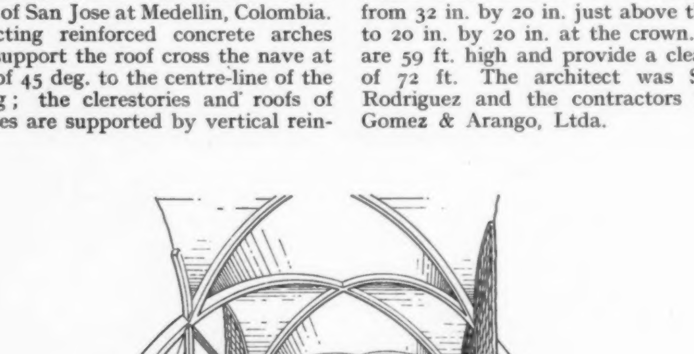


Fig. 3.—Arrangement of Classrooms in a Circular Building.

Construction with Intersecting Arches.

THE illustration shows the frame of the Church of San Jose at Medellin, Colombia. Intersecting reinforced concrete arches which support the roof cross the nave at angles of 45 deg. to the centre-line of the building ; the clerestories and roofs of the aisles are supported by vertical reinforced concrete ties. The arches taper from 32 in. by 20 in. just above the base to 20 in. by 20 in. at the crown. They are 59 ft. high and provide a clear span of 72 ft. The architect was Sr. Nel Rodriguez and the contractors Suarez, Gomez & Arango, Ltda.



Plastic Formers for Hollow Slabs.

Fig. 1.

FORMERS made of plastic reinforced with glass fibres for use in the construction of hollow reinforced concrete floor and roof slabs cast in place and spanning in two directions have been developed by Mills Scaffold Co., Ltd. The formers (*Fig.* 1) are 2 ft. 9 in. square and 9 in., 12 in., or 15 in. deep. A 9-in. former weighs about 10 lb. The formers withstand the loads imposed during constructional work without deformation and are not damaged if they are dropped from a scaffold. Mould oil is not required and, when they are removed from a slab after casting, the formers are free from adhering mortar. The formers fit one inside another during transport.

Close-boarded shuttering is not required, and only timber bearers spanning in one direction and supported on scaffolding (*Fig.* 3) are provided. The formers are laid with adjacent flanges butting together (*Fig.* 2). The spaces between the formers form the ribs and contain the reinforcement. Concrete in the ribs can be consolidated by vibration without the formers moving.

Fig. 2.

A topping of concrete is laid over the formers and ribs. When the shuttering is to be removed the scaffolding and bearers are dismantled and the formers are prised out of position, leaving a cellular soffit to the slab which can be covered by ceiling board or other lining.

Since the formers are intended for slabs 11 in. or more in thickness and spanning in two directions, fewer beams than usual are required; alternatively the slab can carry a greater load if the beams are at

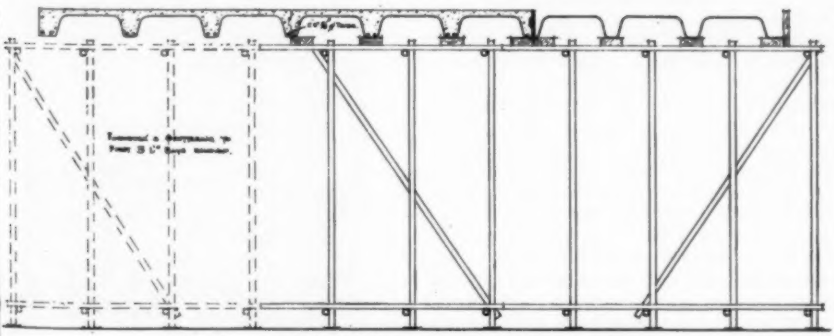
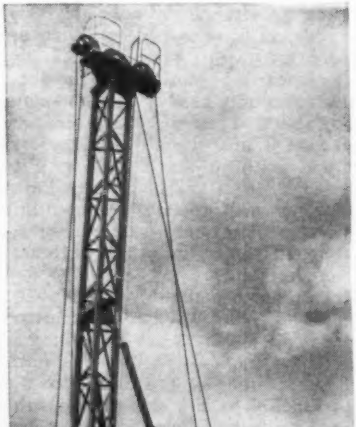


Fig. 3.—Scaffolding for use with Plastic Formers.

normal distances apart. The width of the ribs if the formers are placed close together is 5 in., but if wider bearers are used the formers can be spaced apart so that a rib up to 8 in. wide can be provided; this results in the spacing of the ribs being equal to the greatest spacing recommended for ribbed floors in B.S. Code No. 114 (1957), that is 3 ft. centre to centre.

For floors designed to carry the imposed load recommended for offices in B.S. Code No. 3, Chapter V, an 11-in. slab is suitable for panels up to 30 ft. square, and a 14-in. slab, comprising a mould 12 in. deep with 2-in. topping, for panels up to 45 ft. square.

A New Pile-driving Frame.



Oil jacks and motors control the operation of the winch and the raking, rotating, adjustment of position, travelling, and other movements of the pile-driving frame shown in the illustration. The frame is constructed of steel and can accommodate a hammer weighing 4 tons and a pile up to 6 tons in weight. The height of the frame is 67 ft. and the greatest length of pile which can be driven is about 50 ft. when the point of the pile is at the level of the base of the frame. The weight of the frame with its machinery, but without the hammer, is about 20 tons, but for ease of transport it is dismantled into several pieces the heaviest of which weighs 4½ tons. For the erection of the frame, a 5-tons crane having a 50-ft. jib and a height of 35 ft. under the hook is required.

The power unit, which is mounted on the base, is a diesel engine which drives a dual oil pump which in turn provides power for the oil-operated motor which drives the winch and the jacks which operate the devices for horizontal movement and other adjustments.

Horizontal movement is achieved by means of two inclined jacks attached by hinges to the base, which comprises a number of steel rings the outside diameter of which is 15 ft. 8 in. The frame is levelled by four jacks bearing on the ground below the base. The leaders can be adjusted a distance of 12 in. inwards or outwards, and can be arranged to drive piles having an inclination of 1 in 3 backwards or 1 in 10 forwards.

In the illustration, a McKiernan-Terry diesel-operated hammer is seen suspended in the frame, which was designed by and is supplied by the British Steel Piling Co., Ltd.

Notes on Prestressed Concrete.

The following is an abstract of a paper entitled "How Can the Advantages of Prestressed Concrete best be Utilized?" read by Professor H. Rüsch, of the Institute of Technology, Munich, at a meeting in London arranged by the Prestressed Concrete Development Group.

For some years it was generally thought that nobody was qualified as a good engineer before he had designed a structure in prestressed concrete. About the year 1950, however, the excitement faded, and some engineers began to think that prestressed concrete was very tiresome. The calculations were complicated, and the work on the site was full of risks and needed much care and a large percentage of skilled labour. Some contractors lost money in building prestressed structures. So it became more and more common to believe that prestressing was nothing more than a fashion which had been very interesting but had not fulfilled its promise, and many engineers were ready to say that they had always known that this would happen.

Prestressing will not replace reinforced concrete for all purposes. It is only a new tool which, in the hands of a skilled engineer, offers many new possibilities for concrete constructions which in most cases are not suitable for reinforced concrete. Even so, we should not expect too much. Prestressed concrete has many advantages and many disadvantages. It is not suitable for all types of structure. For example, the lecturer would never design a prestressed umbrella, as many engineers wanted to do. Prestressing can be used successfully only if its scope and limitations are understood and structures designed to make full use of its advantages and avoid its disadvantages.

Temperature Stresses.—Prestressed concrete can be advantageous when it is not possible to calculate the stresses which will occur in a structure. An example is a large salt store from which the salt was to be extracted when required by dissolving it in hot water, and it was impossible to calculate the local temperature stresses which would occur as the solution was drawn off. In such a case a material is needed which has approximately the same strength in tension as in compression and which will withstand very corrosive conditions. If a concrete slab is prestressed in two directions compression results throughout the slab, and amounts to an increase in the tensile strength; almost any equivalent tensile strength can be produced in this way.

Railway Sleepers.—There are many other cases where it is difficult to foresee what kind of stress will occur and where it will occur. An example is a railway sleeper, which should be supported beneath the rails, but very often a completely different state of support occurs —especially when frost is present. In such cases prestressing is the best solution, because the material is prestressed more or less uniformly and will resist positive or negative moments of about the same magnitude. Prestressed sleepers are used widely in Germany; they are economical when they are mass-produced in a factory with machines specially designed for the purpose. An advantage of prestressed concrete sleepers is that they are heavier than timber sleepers; with very long rails without joints, a heavy sleeper is needed to resist high lateral forces.

Cylindrical Tanks.—There are many cases where considerable tension cannot be avoided in concrete structures. In a tie-beam, or in a cylindrical or doubly-curved container, some part is always subjected to very high tensile stresses. If such a structure is in reinforced concrete, it must be designed so as to avoid serious cracking, especially in the case of containers of liquids. The applied tensile stress must therefore be less than the tensile strength of the concrete, which is about one-tenth of its compressive strength. If such a structure is prestressed a very high compressive stress is produced in the concrete, up to perhaps four-tenths of the stress that would cause failure. Therefore the live load must overcome this compressive stress as well as the tensile strength of the concrete before cracks will occur, with the result that the thickness of the wall of a container can be reduced to about one-fifth of that necessary for a reinforced concrete structure.

The easiest way to prestress a cylindrical container is to wind continuous

wire around it under stress. There is the disadvantage, however, that all the wire is on the outside of the wall and must be protected against corrosion. A thin layer of gunite, which is not prestressed, is generally used for this purpose, with the result that when the container is empty there will be no stress in the gunite and when the container is full there will be a high tensile stress which may lead to cracking and so, probably, to corrosion. The centre of gravity of the prestressing steel should preferably coincide with the centre of gravity of the concrete. This is not generally the case in a cylindrical tank; if cracks occur they may be wider than if the steel were in the middle of the concrete and fully bonded with it. Many attempts have been made to place the prestressing wires in the thickness of the wall, but it is difficult to do so, and there is a reduction in the prestress due to friction. It is necessary to use four or more wires or bars to extend around the circumference of the container, and the anchorages and ribs necessary for them result in more expense. The German firm of Dyckerhoff & Widmann has suggested building such a container in sections and prestressing each section independently. This has two advantages, namely, the ribs for anchoring the prestressing steel can be omitted (except one where the work starts), and there is saving in scaffolding and formwork because these can be used several times. A series of containers about 120 ft. high has been built by this method at a sewage works.

Reduction of Deformation.—By prestressing a beam moments are produced of opposite sign to those produced by the load, and thereby the deformation of the member is reduced; therefore, it is best to prestress structures in which rigidity is needed. A notable example is a foundation about 110 ft. long, 80 ft. wide, and about 13 ft. thick for a steel works with a continuous rolling mill on a site in a mining district where subsidence up to 10 ft. could occur. The steel passes at a speed of about thirty-five miles per hour through the last set of rollers, so that a foundation which is completely stiff and does not deform is absolutely necessary. Three separate large foundations were constructed first, and on these hydraulic jacks were placed to support the main foundation. By means of the jacks the level of the main foundation can be adjusted accurately and immediately by pressing buttons. The total weight of the foundation and the machines it supports is about 10,000 tons. The prestressing brought the whole foundation into compression and allowed in advance for any bending which might have occurred in the concrete.

Skew Bridges.—Prestressing can be used to counteract stresses produced by loads in such a way that the statical system of a structure is completely altered. This method has not yet been used enough for its scope to be fully appreciated, but it offers many possibilities. A skew bridge is a type of structure in which this advantage may be used. Such an oblique slab is very difficult to design because little is known about the distribution of the moments. A prestressing cable in the form of a parabola resists the dead load of the bridge. Instead of the need to allow for a complicated distribution of bending moments the solution is simple since the structure is statically determinate. There remains only the problem of calculating the moments due to the live load which, for such slab bridges, are normally much smaller than those produced by the dead load.

Building Frames.—Another example is a building in which beams of relatively large span carry a very heavy load. Shallow beams were required and so they were prestressed, but it would have been costly to prestress the columns. Therefore the beams were prestressed in such a way that the moment produced by the prestress counteracted the moments on the columns produced by the load, thus avoiding tensile stresses in the columns.

"Shell" Roofs.—Prestressing may also be used to reduce stresses which arise near the edges of "shell" structures. A dome in which the horizontal thrust is resisted by a circumferential beam is an example. The shell produces a compression force in the direction of the circumferential beam and, due to the horizontal thrust, the diameter of the ring increases and allows the shell to deflect. Thus, bending moments are produced on the shell. It is unnecessary to allow for bending moments produced by deflection; it is easy to prestress the structure so that no bending moments occur and the shell is in compression only. The circum-

ferential beam must be prestressed so as to produce in it the same compressive stress as occurs in the shell. The diameter of the circumferential beam will be reduced slightly, there will be no bending moments, and there will be no difficulties due to thrust. This is a possibility that could well be exploited in the design of shell structures to permit easier calculation and cheaper construction.

A dome at the University of Hamburg is an example of the use of this method. It has a span of 200 ft. and a very irregular shape, and it would have been difficult to build it in reinforced concrete. By using prestressed concrete, however, the difficulties were greatly reduced. The design would have been complex if mathematical calculations only had been used, so a model was used to study the stress distribution and the best way to prevent bending moments occurring.

Trussed Girders.—Prestressing may also be used to avoid the occurrence of secondary stresses in a trussed girder. Trussed girders were first constructed with pin joints at every junction of the members. This was expensive, so fixed joints were used and the secondary stresses were conveniently forgotten even though it was known that in a reinforced concrete trussed girder they are usually between 150 per cent. and 200 per cent. of the primary stresses. For this reason trussed girders, in spite of their advantages, have not often been used in reinforced concrete structures. By prestressing such girders secondary stresses can be reduced. If some of the members of a truss were subjected to tension and others to compression some of them would be elongated and others shortened, so tending to cause variation of the angles between the members. Secondary stresses arise because the members have to bend to allow the angles at the rigid joints to remain constant. By the use of prestressing, compression may be produced in each member which is otherwise subjected to tension, so that all the members are in compression, and a uniform shortening of all the members occurs. The angles remain constant, and there are no secondary stresses. When a live load is applied secondary stresses appear, but an average state of stress occurs, for example, when the dead load and half of the live load are applied. A prestressed bridge designed in this way is being built to carry a motorway and another road for local traffic over a span of about 330 ft. The bridge is being built without scaffolding, the work being cantilevered from temporary supports. The estimated cost of this bridge is about half of the lowest tender for a steel bridge.

Precasting.—It is a major advantage of prestressing that beams and shell roofs can be sub-divided into parts suitable for precasting. Its use is particularly advantageous in shells, because most of the forces do not act at right-angles to the joints. Prestressing can be used to produce a high compressive force, increasing the friction in such a joint and allowing the forces to cross the joints at an angle; this idea was first used by M. Freyssinet in France.

The use of prestressing makes it possible to vary the stresses in a structure day by day as construction progresses. A bridge was recently built in sections each about 10 ft. long at the rate of a section a week. Scaffolding was replaced by a carriage rolling on the part of the bridge already built. Every construction joint was put in a state of stress by prestressing back to the previous section. The first prestress—about half the full amount—was applied eighteen hours after casting the concrete, and one day later the remainder was applied. The stress distribution during the construction of the bridge altered nearly every day, and the amount of prestress had to be changed accordingly.

Every year new possibilities are discovered and static systems found which can only be used if prestressing is employed. Prestressing is more than a fashion. It is a good method of constructing specific types of structures, but it should not be used indiscriminately to replace reinforced concrete.

Remotely-controlled Measuring Devices.

An electronic device (*Fig.* 1) is now available for measuring strains, deflections, pressures, forces, and temperatures, and it is stated that the measurements are accurate within 1 per cent. The apparatus comprises a transmitter and a receiver. The transmitter contains a wire that can be made to oscillate by excitation from the receiver. The damped natural vibration of the wire varies with the quantity to be measured, and is transmitted to the receiver that contains a wire whose frequency may be tuned to that of the transmitter. The vibrations of the wire in the transmitter are indicated on a cathode-ray tube as a vertical line, and those of the wire in the receiver as a horizontal line. The frequency of the wire in the receiver is adjusted until a circular or elliptical figure is obtained on the screen, indicating that the vibrations are synchronised. The

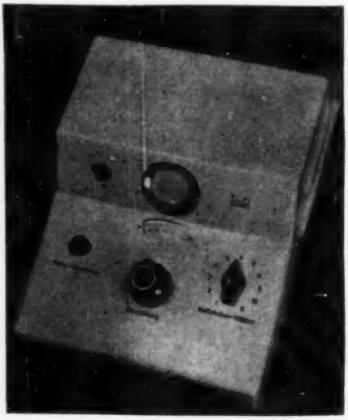


Fig. 1.

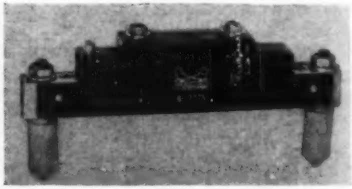


Fig. 2.

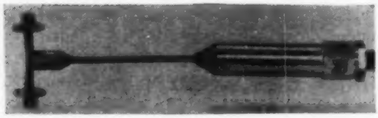


Fig. 3.

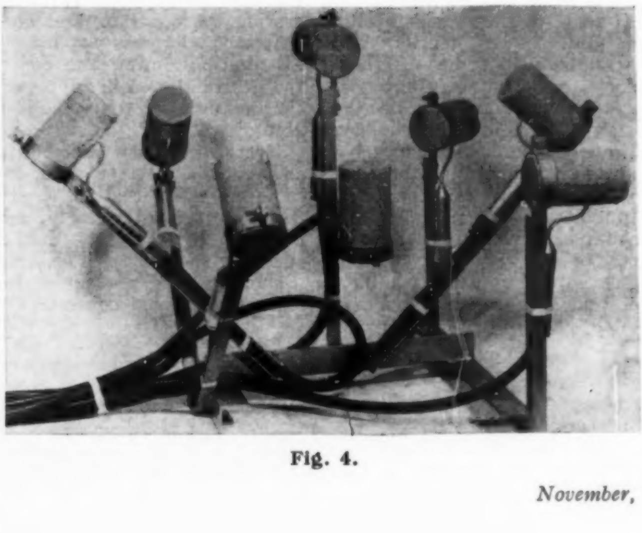


Fig. 4.

adjustment necessary of the transmitter multiplied by a constant is the required measurement.

The transmitters are made for a variety of types of measurement. That shown in *Fig.* 2 is for measuring the strain at the surface of a concrete member. The gauge length is 11·4 cm., and it is claimed to have a sensitivity of 0·0000025 cm. per centimetre of the scale on the instrument.

The transmitter shown in *Fig.* 3 is for measuring the strain in the interior of a structure. It is cast in the concrete, usually after being first cast into a small cylinder of concrete. Eight such transmitters arranged to measure strains in three planes are shown in *Fig.* 4.

These apparatus are made in Hamburg by Messrs. H. Maihak, A.G., who are represented in this country by Messrs. Smail, Sons & Co., Ltd., of Glasgow.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", November, 1909.

Western District Post Office, London.—This number contained the third article describing the design and construction of this reinforced concrete building, which is 190 ft. long, 125 ft. wide, and about 50 ft. high. It is mainly of beam-and-slab construction and was designed by Messrs. Edmond Coignet, Ltd. The following is taken from the article.

"The sorting office measures approximately 141 ft. in width and 137 ft. in depth. The four central pillars support beams which have a span of 45 ft. in one direction and 39 ft. in the other. The areas of floor between principal beams measure about 1800 sq. ft. The area of floor in each corner of the sorting office is composed of principal and secondary beams. The remaining area of floor between the central pillars is suspended to upper beams forming at the same time the inner walls of the courtyard. The two suspended beams which extend throughout the width of the floor without any apparent support are supported by upper beams, as otherwise the span appears to be about 115 ft. The other end of these beams is fixed into the large beams forming the outer walls and containing windows in their lower portion. This supporting slab is 12 ft. high, and is composed of a double framework which is partly a beam with stirrups in the lower portion, and a frame of equal resistance in the upper portion above the floor."

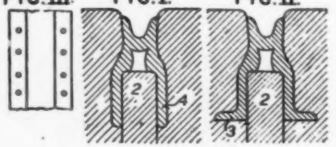
[The building is still in use and is substantially the same as when it was built, but it is proposed that within the next few years the sorting and associated operations will be transferred to new premises now in course of construction in Rathbone Place. The new building is also of beam-and-slab reinforced concrete construction.]

Patent Applications.

Expansion Joints.

EXPANSION joints, particularly for concrete roads, are profiles made of a composition comprising a polymer of isobutylene with a minimum content of 80 per cent. isobutylene and a proportion of at least 30 per cent. of a silicate filler. The

FIG. III. FIG. I. FIG. II.



profiles (4) may be extruded or rolled to the cross-sections shown in *Figs.* I and II, placed on joint boards (2), and concreted in. They may have lateral stabilising surfaces (3), perforated at 10 to 20 mm. intervals as in *Fig.* III.—No. 741,457. Badische Analin- and Soda-Fabrik A.G. September 7, 1953.

Protected Reinforcement.

FOR use in porous concrete, steel reinforcement has a coating of a corrosion protective layer of a tough, non-brittle, hydrophobic material containing grains partly projecting therefrom, the grains being of material having surfaces capable of adhering to concrete.

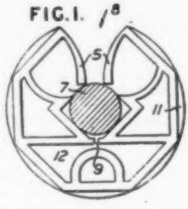
The coating may be applied by first dipping the steel into the liquid coating or in an emulsion or solution of the coating or by spraying thereon, then dipping the steel into the particles or scattering them or blowing thereon, for which warm air may be used.

The protective layer may comprise soluble or fusible plastics, bitumen or rubber compounds, waterglass or soluble intermediate products of phenol and urea formaldehyde. The particles may comprise sand, ashes of heated or burnt slate, various rock particles, or sawdust. The coating may be mixed with a filler such as cement in an aqueous emulsion, sand in linseed oil, sawdust in phenol plastic, or talc dust in cellulose esters. Corrosion-protecting agents such as lead oxide and chromium compounds may be incorporated in the coating.—No. 745,497. Skövde Gasbetong A.B. November 30, 1953.

Bar Spacers.

DISC-LIKE spacers for positioning reinforcement bars are made of a resilient or flexible non-absorbent synthetic resin material and are provided with a flared slot (8) so that they can be clipped on

FIG. I.



to a bar (7) and retained in position on it. An aperture (12) and a slit (9) make the spacer more flexible, and prongs (5) prevent accidental removal. A rib (11) may be replaced by projections or teeth or may extend the full width of the spacer.—No. 715,563. F. W. Berry. August 17, 1953.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Films on Building. Grosvenor Museum, Chester. November 16. 7.30 p.m. Also Five Ways House, Islington Row, Birmingham. November 19. 7.15 p.m.

Formwork Design and Construction. By J. G. Richardson. College of Building, Hatfield Road, St. Albans. November 18. 7.15 p.m.

Good Concrete from Local Aggregates. By R. Cameron. Technical College, Corporation Road, Redcar. November 18. 7.15 p.m. Also at Municipal College, Burnley. November 19. 7.15 p.m.

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